THE ITÁ HYDROPOWER PLANT ON THE URUGUAY RIVER

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1. INTRODUCTION

Itá Hydroelectric Plant is situated on the Uruguay River between the towns of Itá in the State of Santa Catarina and Aratiba in the State of Rio Grande do Sul, with geographic coordinates of 27º16'S and 52º23'W (see Figure 1).

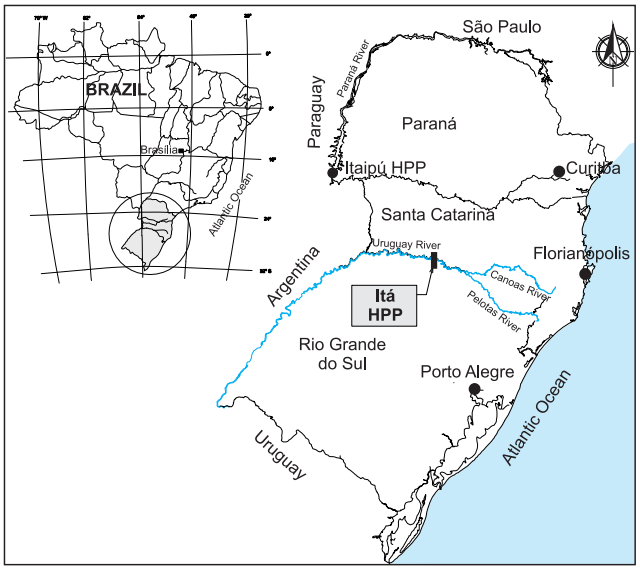

Figure 1 - Location of the Project Site

This project was the first in the Brazilian energy sector when it established a model based on the legal benchmarks available, in which ENGEVIX prepared the privatization model and bidding process, as well as all documentation relating to the turnkey contract. Photo 1 shows an aerial view of the project.

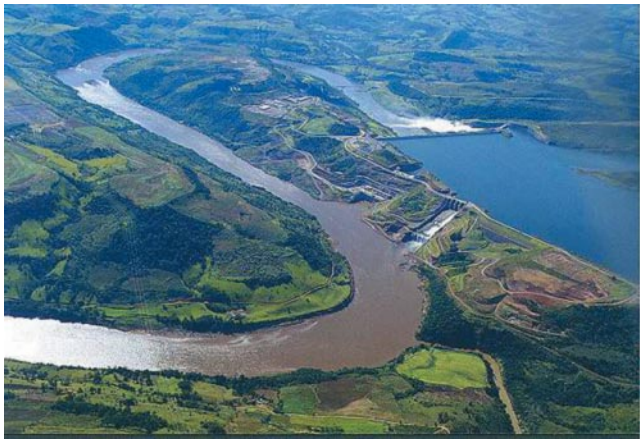

Photo 1 - Aerial View of Itá HPP

Itá Hydropower Plant has a capacity of 1,450 MW and includes the first large concrete faced rockfill dam built in the Uruguay River basin, around 125 m in height and 880 m in length, with a volume of 8.9 million m³. The works for implementing the plant began in March 1996 and commissioning of the first generating unit was in April 2000.

Power generation and transmission concession at present belongs to the group of companies Tractebel Energia, CSN and Itambé. This plant was built under a turn-key project, headed by CBPO (civil works), TENENGE (assembly), ABB/ ALSTOM / VOITH / COEMSA-ANSALDO / BARDELLA (electromechanical equipment) and ENGEVIX (design).

2. DESCRIPTION OF PROJECT

The sole purpose of the project is electric power generation. The general layout of the design, shown in Figure 2, takes advantage of the fact that the river flows in a long U bend called the Uvá bend, where the narrowest point offers an excellent site for the generation circuit and diversion tunnels.

The plant has a generating circuit equipped with five Francis turbines, each with a 290 MW capacity, and enjoys a gross head of 105 m. The generating circuit structures consist of a gravity concrete intake, followed by five concrete-lined penstocks 200 in length and 8 m in width.

The plant also has two spillways controlled by gates with the capacity to evacuate the probable maximum flood of 50,000 m³/s. The spillways are equipped with ten radial gates 18 m wide and 20 m high, of which six gates are in the main spillway and four in the auxiliary one.

The CFRD dam is 125 m in height, which provides a total 105 m head for power generation in a site with a mid to long term flow of 1,100 m³/s. The total volume of the dam is 8.9 million m³ of compacted rockfill and transitions. Three embankments with earth and rockfill section were built, 20-30 m in height and 400-500 m in length, to fill topographic saddles, in some places restricting the reservoir area.

The river diversion includes five horseshoe section tunnels, two with a control structure fitted with trashracks and floor at El. 258.00 m, and three with the inlet floor at El. 268 m for operation during the heaviest floods.

The high flood peak on the Uruguay River was the reason for building a 51 m high cofferdam, conceived for a ten-year recurrence interval (flows of up to 19,000 m³/s). The first stage of the main dam (88 m in height) was planned

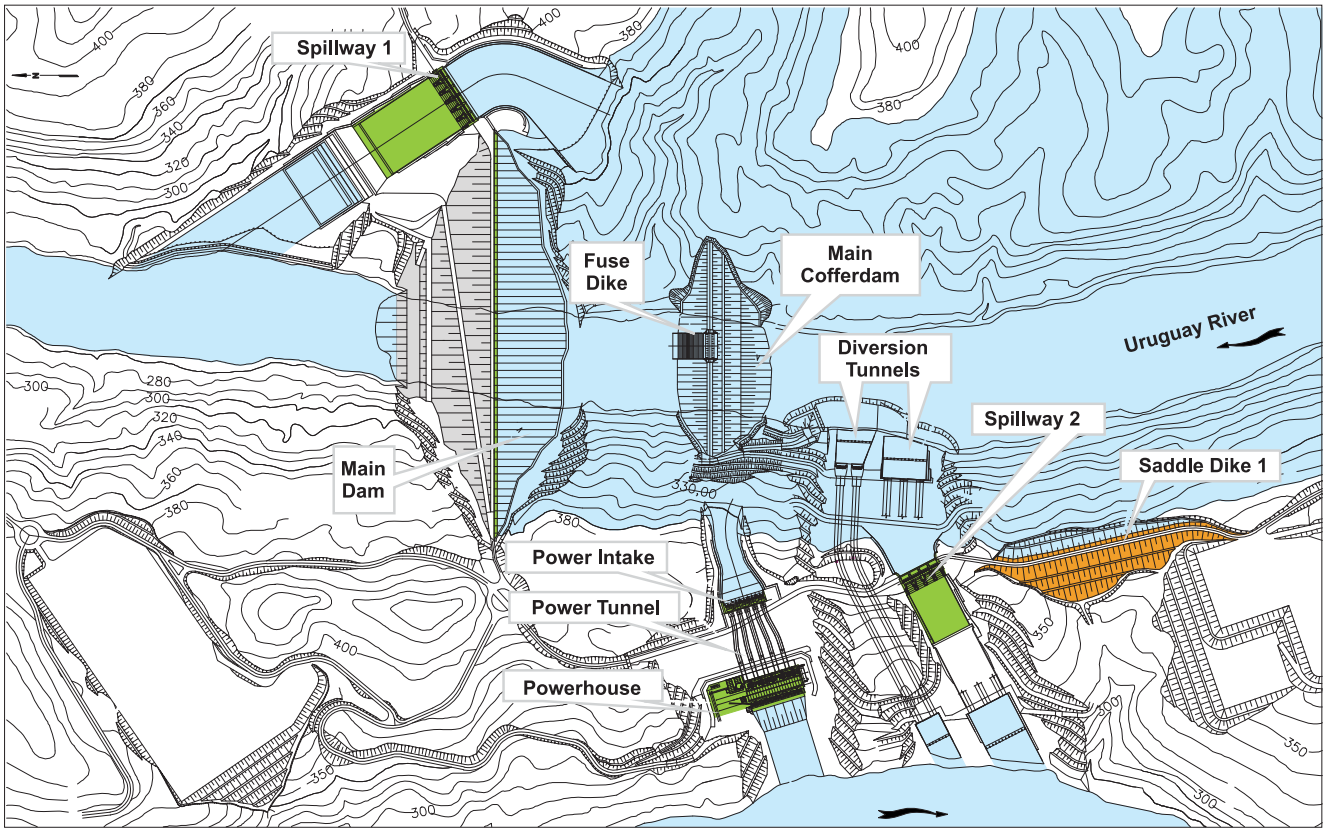

Figure 2 - General Layout of Plant

for a recurrence interval of 500 years or 39,000 m³/s of the peak flow.

The main quantities involved in the undertaking are listed below:

- Common excavation 9,600,000 m³
- Rock excavation 8,000,000 m³
- Underground excavation (5,000 m of tunnels) 642,000 m³
- Rockfill and transitions 11,110,000 m³
- Conventional concrete 504,000 m³
- Steel/cement 30,000/115,000 ton

3. GEOLOGY, GEOTECHNICS AND FOUNDATIONS

On the project site the Uruguay River has a width varying between 200 and 250 m and average depth of 10 m for the normal water level. The river carves through sub-horizontal basalt flows to form not too deep a valley with difference in level between the top of the plateau and valley bottom, of around 160 m. The soil overburden on the abutments is generally quite thick, deeper on the downstream side of the region where the river bends (Uvá curve). The riverbed is essentially rocky and the slopes have varying gradients of 20º to 30º, sometimes with two terrace levels in the lower parts of the right abutment.

The project is built in the Paraná Sedimentary Basin, in the Serra Geral Formation, consisting of basalt flows. Nine basalt flows were found between elevations 200 and 400, called D to L in descending direction. These flows are sub-horizontal, with a slight general dip of 0.5º to the NW, and may present dips of up to 25º as exceptions in some local stretches. The thickness of such flows varies between 10 m and 50 m.

One of the outstanding geological characteristics in the Itá region refers to the thick weathering overburden at the job site, formed by a saprolitic colluvial mature residual soil and soft weathered rock. The tops of the watershed divides have typically 5-10 m thick weathering cover and slopes of 10-20 m. In local stretches, especially along the structural alignments, these thicknesses may exceed 30 m.

The alluvial deposits normally occur in the terraces on the slopes, sometimes with a large quantity of blocks, forming bodies of talus. The alluvia are restricted to the convex banks and flood plains at the mouth of the tributaries. The main geological conditioning factor in determining the dam axis was the thickness of the weathered overburden where the plinth will be installed. This means that the foundation of the dam plinth crosses a region with shallow cover on the right bank below El. 315.

In the riverbed stretch, the main conditioning factor refers to the sub-horizontal discontinuity J. This feature between elevations 240 and 250 corresponds to a more fractured zone, with weathered materials between the fractures and high permeability.

During construction, there was an unexpected

geological finding when high residual stresses in the rock mass were discovered, at that time unprecedented in hydropower plants built on basalt rock in the Paraná Basin. Spalling phenomena of the rock were observed in the Itá underground excavations, in the "I" and "J" basalt flows.

Shortly after the start up of the plant, two erosions occurred in the unlined chute of the auxiliary spillway despite the good quality of rock mass and extra care taken with the excavations. This process may have been caused in the zones peripheral to the discontinuities in the area, beyond the strip immediately above the contact between flows at El 294, which acted as relief levels of underlying blocks, aggravated by the pre-existing spalling deriving from relief of stresses in the rock mass. Accordingly, the result of this domino effect was that new blocks were eroded as the blocks began to loosen in series. The phenomenon worsened as a result of the flat sub-horizontal layout of these litho-structural features, in addition to greater susceptibility to water erosion of the thin sub-horizontal plates, which after removal caused another redistribution of stresses in the rock mass, back-feeding the erosion process.

4. HYDROLOGY, HYDRAULICS & ENERGY STUDIES

The Uruguay River basin at the site of the Itá HPP covers a drainage area of 44,500 km².

The average flow for the period under observation (1958-1991) is 1,080 m³/s. The daily minimum flow of 114 m³/s occurred in March 1968, and the daily maximum flow of 29,620 m³/s occurred in August 1984.

The Uruguay River at the project site varies in width between 200 and 250 m and average depth of 10 m in relation to the normal water level.

The energy studies permitted the definition of the operating conditions, number of units and installed capacity, which resulted in adopting five Francis units with 294.4 MW under a head of 102 m, with total installed capacity of 1,450 MW.

5. MAIN STRUCTURES

The plant layout consists basically of the following:

• River diversion by means of five tunnels in a horseshoe section, two of them (section 14 m x 14 m) in a concrete control structure fitted with trashracks, and another three (15 m x 17 m) at higher elevations during the major flow operation, closed by the cofferdams and plugs to protect from flows of no more than 1,500 m³/s;

• Concrete face rockfill dam with crest at El. 375.50, with a height of 125 m, length 880 m and total volume estimated at 8.9 million cubic metres;

• Two spillways controlled by gates with capacity to evacuate the probable maximum flood corresponding to a flow of 50,000 m³/s, with ten gates, 18 m wide by 20 m high;

• The generating circuit consists of a gravity concrete intake, followed by five concrete-lined penstocks 200 m in length and 8 m in width. The powerhouse, with access yard at El. 294.00 is covered, consisting of five blocks for the Francis generator units and the assembly area, with total length of 52 m and maximum height of 31m. The gas-insulated shielded substation is on the left bank, on a plateau at El. 309 m.

• Single-bus drive substation and two transmission lines as far as the substation of the system, with around 1.8 km in double circuit, with rated voltage of 535 kV.

5.1. River Diversion and Control

The river diversion has always been considered a crucial question in Itá, since the combination of basalt soils with low permeability and steep gradients of the deforested slopes, predominant in the drainage basin, caused flood peaks that responded rapidly in the wet seasons.

A number of preventive measures were taken to guarantee the security of the dam construction during the design stage [Antunes et al., 1991[1]], such as:

• Placing on the abutments around $3x10^6$ m³ of rockfill before diverting the river, strangling the riverbed for around 70 m.

• The river diversion, at the end of the wet season and the increase of the dam in the next six months to protect against the 500-year flood recurrence interval before the start of the next rainy season (El. 348 m). Around 2.2×10^6 m³ of material had to be placed during this period.

• A cofferdam designed for 10-year recurrence interval could protect the works during the first few months after the diversion.

• A fuse-plug spillway and flood channel were planned to protect the cofferdam should the water rise during the first rains after the diversion, when the dam would be well above this level, but the work would be done in the plinth area.

• A layer of fine rockfill was laid to partly protect the dam construction against failures in case of seepage.

In addition to these preventive measures, a typical hydrological conditions caused by the El Niño phenomenon, which occurred during the works, caused corrective measures to be adopted urgently.

The river diversion through the tunnels was completed on 24 September 1997 (see Figure 3). The rockfill placement for closure was carried out in a slow process, since the river flow at that moment was relatively low (700 m³/s). The operation took 15 hours, during which time around 10,000 m³ of rockfill was laid. The critical phase was when a 15 m wide breach in the riverbed was reached, with a flow of 350 m³/s passing through the breach to a differential of around 2.7m. The maximum opening at the end of the closure was 3.4 m.

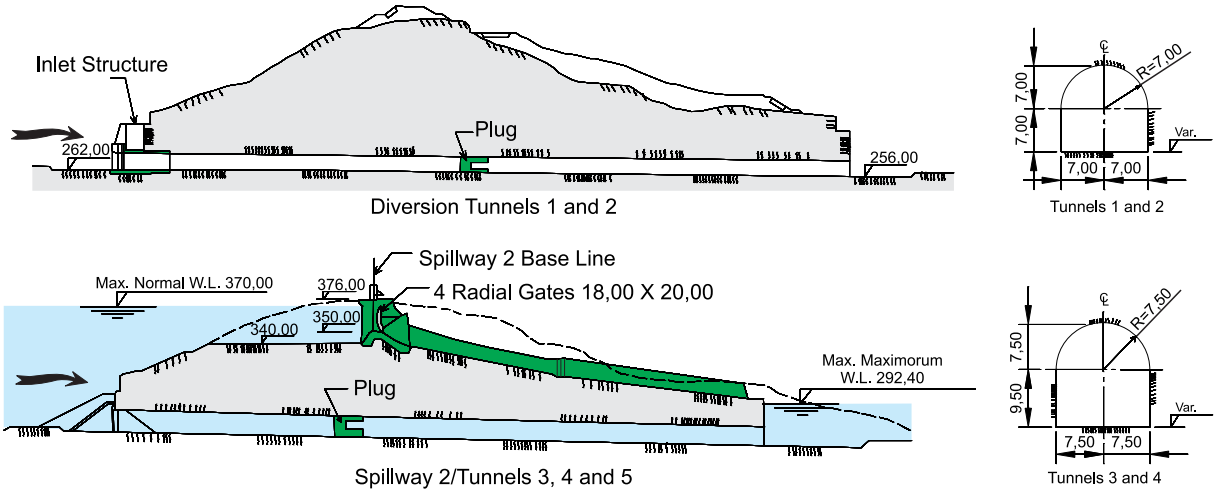

Figure 3 - Typical Section of Diversion Tunnels

The diameter of the largest dumped blocks varied between around 0.9 and 1.2 m, with a volume of 4,000 m³. The water depth at the start of closure was 7 m.

A major characteristic in the process was the simultaneous dumping of material from both banks, always placing large blocks upstream, with a slight upstream slope (reducing the bank) resulting in downstream turbulence.

The entire operation was easily performed because the river had been previously strangled to 30 m and part of the rockfill was stored in the cofferdam to fill some metres above water level. The rockfill, therefore, was pushed directly into the water.

Because of the uncertainties relating to the El Nino phenomenon, which was already causing flooding in other South American countries, and simulations carried out by ENGEVIX, a decision was taken immediately after closing off the river to accelerate the cofferdam construction. On the 16th, around 500,000 m³ of rockfill was placed, partly protected by transitions and a blanket.

A few days after the closure, the constant rain along the basin kept the flows in the 3,000-4,000 m³/s range. Next, between 10 and 12 October, around 160 mm of rain fell along the basin, causing the worst flood ever recorded in October, and consequently closing the river off would hardly be possible before mid-December.

The cofferdam was exposed to 40 m head where the top 20 m was covered only with one transition and relatively high permeability. Furthermore, the operations for the rockfill compaction were changed in order to speedily raise the embankment in the part where a flood-channel was built since it was the narrowest point.

The priority was to seal the critical seepage points above El. 280 m, especially in the narrowest section, resulting in a higher hydraulic gradient (1H: 0.45 V). Very heavy blocks were not placed on this slope because the reinforced rockfill and flood channel would be built over them later.

Leaks from smaller unsealed points on both abutments were not a major concern during the crisis, since the structure of the lower section of the rockfill was strong enough to withstand high gradients, thanks to the interlocking of the large blocks.

The measurements included the following:

- Underwater dumping of a large amount of rock transition with a high degree of fine material (T4) in the central zone and simultaneously on both banks;
- Construction of rockfill stabilisation on the downstream bank;
- Raising the cofferdam by around 3 m.

The extensive works proved to be effective and the major seepage was finally controlled.

The discharge capacities of the diversion tunnels were well estimated, as defined by means of cross checking the upstream rating curve with the downstream one, determined from test results of the small scale hydraulic model.

The upper tunnels were inspected after the October 1997 flood and only slight damage was found, as a result of the high outflows.

Maximum velocities in the tunnels were estimated between 14 and 15 m/s. The average overbreak measured in the tunnels was 0.35m, resulting in a Manning coefficient of roughness (n = 0.0285) for the theoretical excavation line.

During the diversion period, the tunnels began operating with high outflows and no problems, despite a flood of the same magnitude in April 1998.

5.2. Dam

The typical cross-section and zoning of the main dam are given in Figure 4. Curves of classification, densities and permeability of the different materials used in the dam are shown in Figure 5.

A stiffer zone was provided behind the face slab with a 7 m width of a well graded crushed transition T1 (3 to 2 m) and tunnel muck fine rockfill E0 (5 to 4 m), of which plenty was available at the site. This was

compacted in 0,4 m layers, and was followed by a 23 m wide strip of basically sound basalt E1 material, compacted in 0,8 m watered layers.

Close to the perimetral joint, the T1 transition was widened (8 m). In this area the materials were moistened and compacted with six passes of a 90 kN vibratory roller.

The upstream third of the dam was compacted in 0.8 m layers and the rest of the dam in 1.6 m layers. A mixture of sound rock, breccia and vesicular basalt was perfectly acceptable at around 30 m from the face. Preference, whenever possible, was given to the application of sound rock upstream, but a considerable quantity of rock mixture was available, as a result of the blasting of various thin basalt flows.

The compressibilty module of the sound basalt E1 (layers of 0.8 m) reached an average of 40 MPa, and for E2/E3 (1.6 m layers) was 30 MPa. The densities in general were always above 2.0, even for basalt breccias.

5.3. Spillway

The outflow system consists of two spillways, the first being the main spillway, located on the right abutment of the dam. The second, called auxiliary spillway, located on the left abutment, on the top of the diversion tunnels.

The main spillway is of the surface type (see Figure 6) and in its control structure has six radial gates 18.00 m wide by 20.00 m high, with dividing pillars 4.50 m, resulting in a total width of 130.40 m. Its sill was set at El. 350.00 m. The top of the structure of this spillway is at El. 376.00 m.

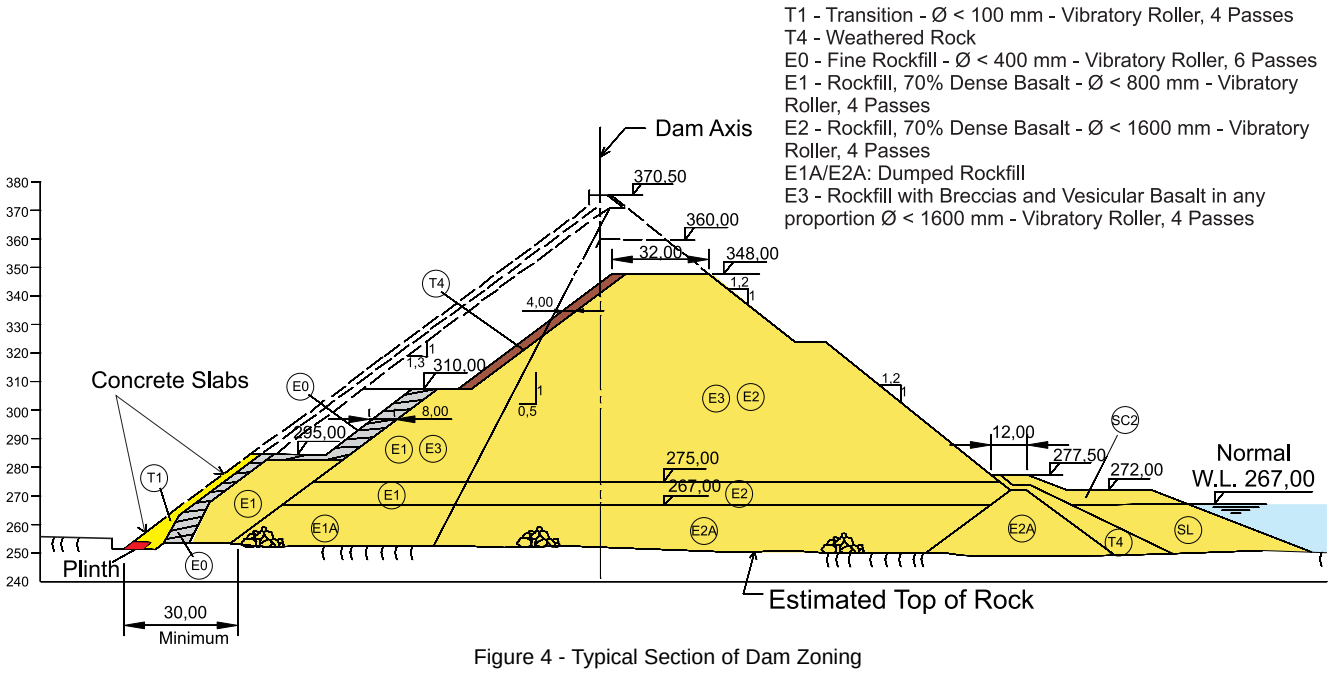
T1 - Transition - Ø < 100 mm - Vibratory Roller, 4 Passes
T4 - Weathered Rock
E0 - Fine Rockfill - Ø < 400 mm - Vibratory Roller, 6 Passes
E1 - Rockfill, 70% Dense Basalt - Ø < 800 mm - Vibratory Roller, 4 Passes
E2 - Rockfill, 70% Dense Basalt - Ø < 1600 mm - Vibratory Roller, 4 Passes
E1A/E2A: Dumped Rockfill
E3 - Rockfill with Breccias and Vesicular Basalt in any proportion Ø < 1600 mm - Vibratory Roller, 4 Passes
Dam Axis
370,50
360,00
348,00
32,00
4,00
310,00
295,00
8,00
275,00
267,00
277,50
272,00
12,00
Normal
W.L. 267,00
Concrete Slabs
Plinth
30,00
Minimum
Estimated Top of Rock

Figure 4 - Typical Section of Dam Zoning

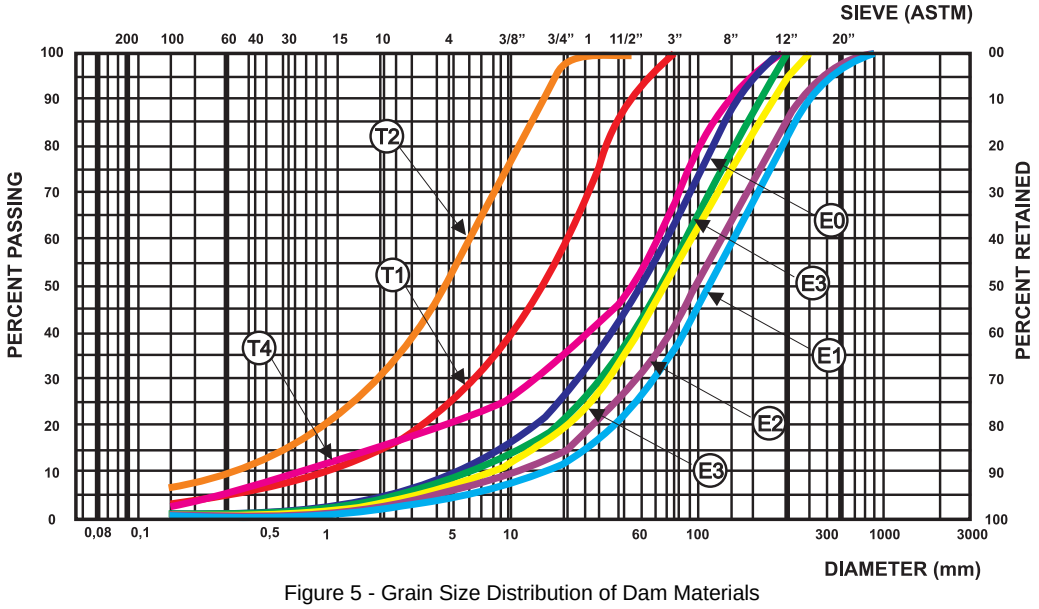

Figure 5 - Grain Size Distribution of Dam Materials

The spillway chute downstream from this control structure was concrete lined over 195.05 m in extension. Its initial point is at El. 337.64 m and its slope is 19.65%. At the end of this chute is the deflector for launching the jet of water into the plunge pool. This plunge pool is around 104 m downstream from the deflector, with its bed at El. 245.00 m, outlet sill at El. 265.00 m, 70 m in length and totally excavated in the rock. A platform of 37 m in length between the deflector and plunge pool is at El. 300,00 m.

The purpose of this plunge pool was to reduce the water flow velocities and oscillation in the water levels next to the downstream slope of the dam and the river banks downstream from the dam.

The auxiliary spillway is also of the surface type and in its control structure has four radial gates 18.00 m wide by 20.00 m high, with 4.50 m dividing pillars, resulting in the total width of 85.50 m. Its sill was set at El. 350 m. The top of the structure of this spillway is at El. 376,00 m. The typical section is shown in Figure 7.

The spillway chute downstream from this control structure with its starting point at El. 337.64 m and slope of 19.6% was concrete lined over an extension of 120 m. In the rest of the downstream stretch over 167.90 m with slope of 12.50%, the bottom of the chute was merely excavated in rock. The end of the this spillway's chute at El. 294.00 m coincides with the outlet section of the top tunnels and the final launching of the jet of water goes directly into the outlet channel of these tunnels at El. 254.00 m.

5.4. Headrace Channel and Intake

The intake consists of a hollow gravity structure, with maximum height of 35.50 m and downstream face slope of 1V:0.18 H. The intakes are independent for the five generator units and each of the blocks has two openings because of the central pillar to support the trashracks. Each opening is fitted with two lines of slots for the gates, the downstream one corresponding to the emergency gate and the upstream to the stoplog.

The fixed wheel gate is designed to protect its generator unit with an emergency closing of the intake, under any flow conditions of the turbine.

5.5. Penstocks

The penstocks were excavated in rock and concrete lined. In their final stretch the penstocks are steel lined, designed to withstand the stresses from the transitory regimes of the units.

The excavation of the stretch that starts at the intake,

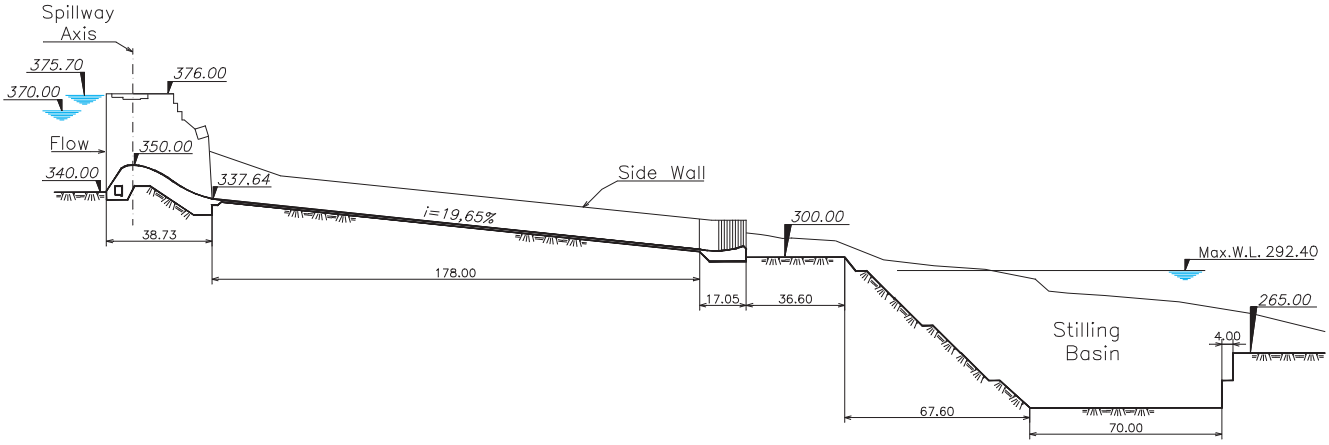

Figure 6 - Longitudinal Section of the Main Spillway

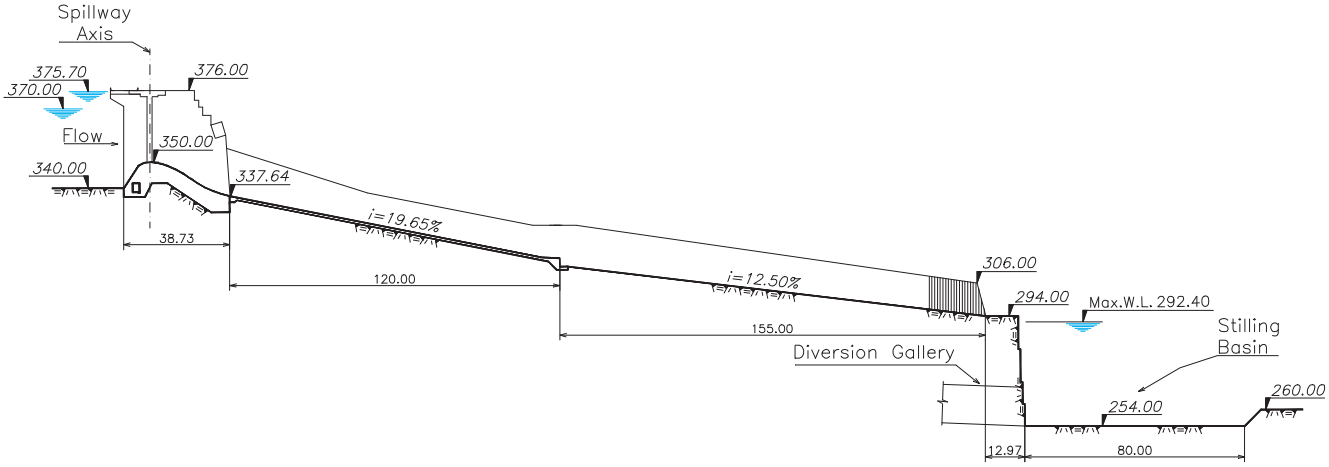

Figure 7 - Longitudinal Section of Auxiliary Spillway

152.35 m in length, has a circular section with radius of 8.00 m and length of around 155 m. The armoured stretch has a horse-shoe section with diameter of 7.50 m and length of 44.65 m and the following stretch as far as the spiral box with 18.35 m in length has a circular section with diameter of 6.80 m.

5.6. Powerhouse

The indoor powerhouse, with five blocks for the generator units, each 23.50 m wide lengthwise and 38.2 m internal width crosswise, houses five Francis vertical axis hydraulic turbines, with rated capacity of 294.40 MW and rated velocity of 128.57 rpm, adapted for direct coupling to three-phase synchronous generators of 305 MVA. Figure 8 shows the generation circuit.

The maximum height of the structure is 53.50 m, measured from the foundation of the elbow of the draft tube at El. 240.50 m, to the transformer yard at El. 294.00 m. The assembly area, with floor at El. 270.30 m, is an extension of the machine hall, consisting of two blocks situated on the left of the generating units, with 51.85 m in width lengthwise.

The loads inside the structures are moved with the help of two overhead travelling cranes with unit capacities of 3600/600/50 kN, a rail track at El. 294.00 m and span of 20 m.

The electromechanical equipment is laid out in four internal halls to the powerhouse, situated at elevations 286.10 m (mechanical tunnel), 278.20 m and 270.30 m (lower and upper electrical tunnels) and 260.70 m (electromechanical tunnel), in addition to the transformer platform at El. 294.00 m. Vertical access to the tunnels is by means of three stairs, in blocks 1, 3 and 5, or by lift in block B of the assembly areas connecting the control building to the lowest level in the powerhouse.

The dewatering and drainage sumps are situated to the right of the generator units in block B of the assembly area. The pump room of these wells is above them with floor at El. 242.45 m.

Access to inside the powerhouse is through a reception area situated in block B of the assembly area over the tunnels, at El. 294.00. Next to the reception area there is access to the top tunnel using a lift and stairs, as well as to the control building. The tailrace is fully excavated in rock, with a width of 96.00 m.

6. CONSTRUCTION

Special attention was given to seepage control provisions to protect the partially built dam (first stage at El. 348 m) in case of a major flood (up to 1:500 years), since the concrete face slab and T1 transition were not in place. It was found that the rockfill E0 had not enough fines to lower the piezometric line. Photo 2 and 3 illustrate construction stages of the concrete-faced rockfill dam. The solution was to protect the slopes, some 35 m behind the upstream slab line, corresponding to the first construction stage, from El.348 m to El.310 m, with a 4 m-wide zone of highly weathered rock (T4) with 25 per cent of the material passing mesh n°.4, compacted in 0,4 m layers.

A remarkable advance in the CFRD construction process, conceived in Itá, was the development of an extruded concrete curb wall under the slab face. This is a mixture of aggregates of less than 0.75 inches (19 mm) with 75 kg/m^3 of cement, laid on the upstream face. This is a fast and economic solution to stabilizing the face, substituting the former solution of asphalt emulsion with sand and roller compressors.

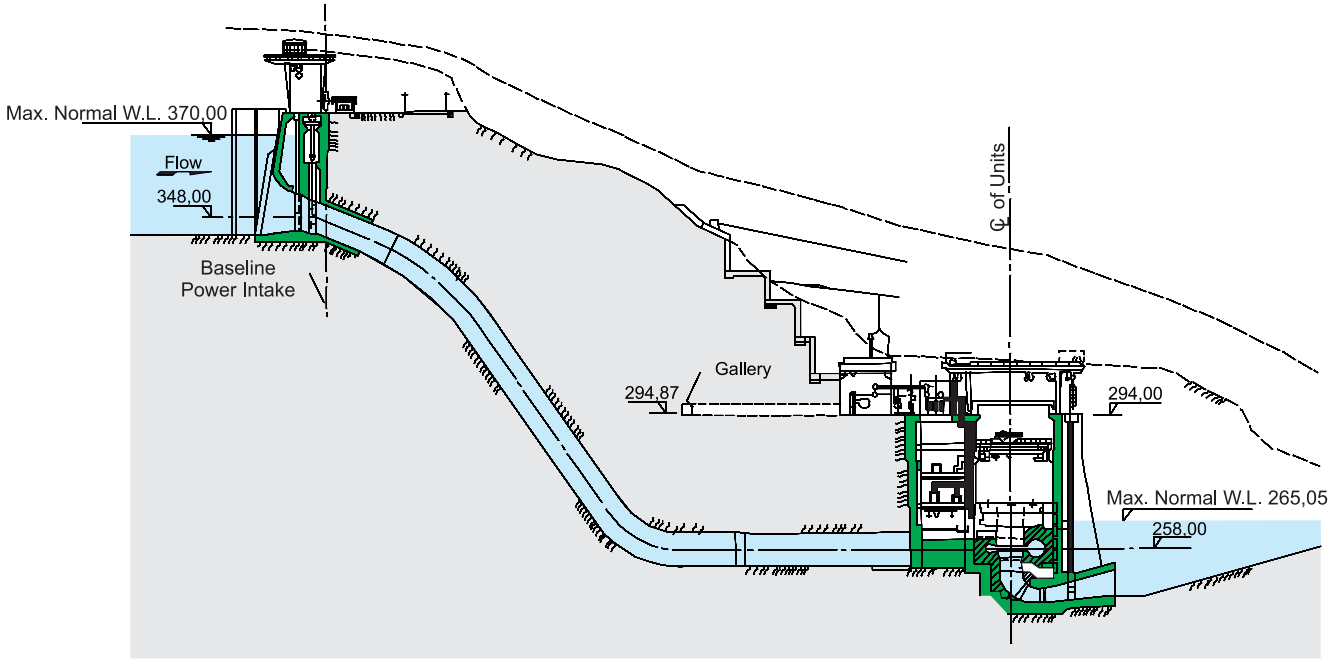

Figure 8 - Generation Circuit

The extruded curb wall provides better protection for the dam in cases of flooding, more safety for the personnel working on the plinth, a more regular surface for the slab, with consequent reduced grading, with good concrete fixing and better sliding of formwork.

A particular feature of Itá was the plinth design. Major excavations were made to reach the top of only slightly altered to sound rock and it was necessary to build gravity concrete walls for grading the ramp along its course. The stretches with major differences in level associated with the layers of basalt breccia were treated with shotcrete and anchor bars. Along this structure consolidation holes 4 m deep were drilled and grouted, plus a watertight grout curtain with varying depth depending on the hydraulic head in the stretch under consideration. The average of cement take was 20 kg/m for consolidation grouting and 56 kg/m for exploratory boreholes.

Photo 2 - Start of Dam Construction before River Diversion

In the dam area blasting was carried out only to install the plinth. Cleaning the foundation to the rock top covered one third of the upstream area, with occasional isolated pockets of weathered rock. This cleaning was more thorough in a 25 m stretch adjacent to the plinth at the base of the dam, diminishing to 10 m at its top. In the remaining area, around two thirds downstream, the foundation consisted of a hard saprolite, with SPT counts higher than 15 blows.

Photo 3 - Overview of Dam Construction

The construction of Itá HPP began in 1996. On 1st March the machines began to lay the rocky material on the bed of the Uruguay River to build a cofferdam.

On 15 May 1997 concrete pouring in the powerhouse area began. A month and a half later the electromechanical equipment began to be assembled. On 24 September, the Uruguay River was diverted through five tunnels excavated in rock.

On 15 December 1999 the last diversion tunnel of the river was plugged, and the reservoir started to be filled, which in a short time occupied an area of 141 Km².

In June 2000, the Itá Hydropower Plant began to actually generate. The second machine began commercial generation on 28 August, one month before the date stipulated in the contract. The start up of the other machines was also substantially advanced. In 2001 the works were completed and the Itá hydropower plant was now running fully.

7. ECONOMIC, ENVIRONMENTAL AND SOCIAL ASPECTS

The environmental impact study (EIA) of Itá HPP, an integral part of the Environmental Impact Assessment (AIA), was developed by the National Consortium of Consulting Engineers (CNEC) in 1989 and covered the entire area of influence of the project. The Itá HPP RIMA (Report on Environmental Impact) was prepared by the same Consortium and presented for appreciation of the community in 1990.

Environmental control measures were adopted in order to keep the environmental condition of the ecosystem in the region of influence of the Itá HPP project and to help improve it. This attitude is reflected in carrying out environmental programs, which range from complying with the laws and regulations of the relevant agencies, and confirm the engagement with the environmental issue.

With the decision of the IBAMA (Brazilian Environmental Authority) to transfer to state agencies the definition of the management category, it was decided to create a State Park in Santa Catarina and a Municipal Park in Marcelino Ramos in Rio Grande do Sul.

In both units, flora and fauna studies were undertaken in which the following aspects are worth mentioning: mapping the vegetation typologies, using GPS to locate matrix trees (seed bearing), identifying and following up the dispersion of the invading exotic species, surveying tree species with main ecological characteristics (phenology, morphology, dispersion, etc.), follow-up and transplant of endemic species (Dyckia distachia); monitoring the natural regeneration development to check if there is need for intervention. The study of the fauna included: setting traps to identify the existing fauna, identifying the habitat of the main species, study of populations and composition of classes and other details in the management plan.

A private entity of públic interest was created in order to take care of the management of the Fritz Plaumann Park.This organization provided the incentive to extend the monitoring and refurbishing structures of the research centre. Supported by the Consortium Itá's incentive, improvements are being implemented in the park's structure and actions for its smooth functioning. Highlights include the preparation of ecological trails, observation points, installing signs, improving visiting places and so on.

Local universities and researchers are taking sustainability actions in the park. Some worth mentioning are the monitoring of the fauna and also of the proliferation of exotic species, especially Hovenia dulcis (Japanese raisintree) that, due to its high disseminating power jeopardizes the development and regeneration of the native species.

Under Decree no. 1994, dated 28 September 2005, the Severiano de Almeida town government created the Municipal Environmental Preservation Park (PMPA) of public domain, mainly in order to protect the natural resources, flora and fauna, and to reconstitute the gallery forest. The land on which the PMPA was implemented covers an area of 153,872 m² and was donated to the Severiano de Almeida county by the Itá Consortium.

The socioeconomic aspect was another parameter considered when establishing the forest protection strip of Itá HPP, bearing in mind claims and aspirations of the Movement of those Affected by Dams (MAB, formerly CRAB), in order to minimise the impact on the populations that occupy the areas to be vacated. The line was established as expropriation elevation corresponding to a strip of at least 30 m measured from the normal maximum level of the reservoir (El. 370 m), all along the perimeter of the reservoir and to the elevation corresponding to the flood of a hundred-year recurrence interval, the most external line being adopted. Accordingly, a strip of forest protection was created around the water line.

8. PERFORMANCE OF THE PROJECT

The Itá Hydropower Plant has a complete surveillance system. The monitoring instruments are distributed in the concrete structures: powerhouse, intake and spillways; in the embankments: dikes and rockfill dam. Its measurements generally cover measures of deformations, stress, neutral pressure, temperature and seepage flow.

The concrete structures and powerhouse have the following instrumentation installed: rod extensometers, mechanical tri-orthogonal gauges, electrical joint gauges, direct pendulum, no-stress boxes, thermometers, Casagrande piezometers, flow meters, observation wells and surface benchmarks. Rod extensometers, flow meters, Casagrande piezometers and observation wells are installed in the spillways. Only Casagrande piezometers are installed in the intake structure.

The instruments installed in the embankments are: Casagrande piezometers, flow meters, observation wells and surface benchmarks.

Itá dam has three main sections with instrumentation. These sections are monitored using various instruments, as follows: magnetic settlement gauges and settlement cells (Swedish box) to measure the settlement; and multiple rod extensometer KM type to measure horizontal displacements. In addition to these instruments there are surface benchmarks spread throughout the dam that help follow up both vertical and horizontal displacements, and a triangular weir flow meter for controlling the seepage flows. The concrete facing is monitored by electro-levels, electric jointmeters and triorthogonal jointmeters. Figure 9 show the general location of these instruments.

These instruments provide satisfactory results and have been in operation for approximately eight years. Today their behaviour shows a tendency to stabilisation, only slight settlements and horizontal deformations remaining from the slow deformation process. The vertical deformation values presented an increase of only 5% in relation to the deformation measured on completing the filling.

The largest settlements noted occurred in the section on the river bed (32+8.20 m), close to half the dam height, due to a favourable combination between the underlying layer and the pressure due to the overlying fill. At the end of the construction period the largest settlement observed was 3,225 m in CR-10 installed at this benchmark, in the central region of the embankment.

The deformability module values referring to the time of Itá's construction were close to 50 MPa for the upstream zone (compacted materials with no more than 80 cm) and 25 MPa for the downstream zone (compacted materials of 100 cm to 160 cm).

The multiple rod extensometers, located in the upstream area of the rockfill embankment, which during the construction stage indicated an upstream displacement, during the reservoir impounding, began an inversion process in the displacement direction towards downstream. There is only a slight variation in displacement between the reservoir filling and today, which shows that the dam is attaining a steady state.

The surface benchmarks installed in the crest of the dam behave quite similarly to that of the instruments installed inside the dam embankment in which the most significant displacements occurred at the end of the construction and start of the filling, and now these instruments show a tendency to stabilise.

Deformations from settling and significant settlements occurred on the concrete slab of the upstream face during the construction period. The horizontal displacements are characterized by their presenting a deformation of the shells to the centre of the structure in the top half of the dam, causing a narrowing, while the bottom half moves in the opposite direction, causing an opening in the base. During the filling period, however, the hydraulic head on

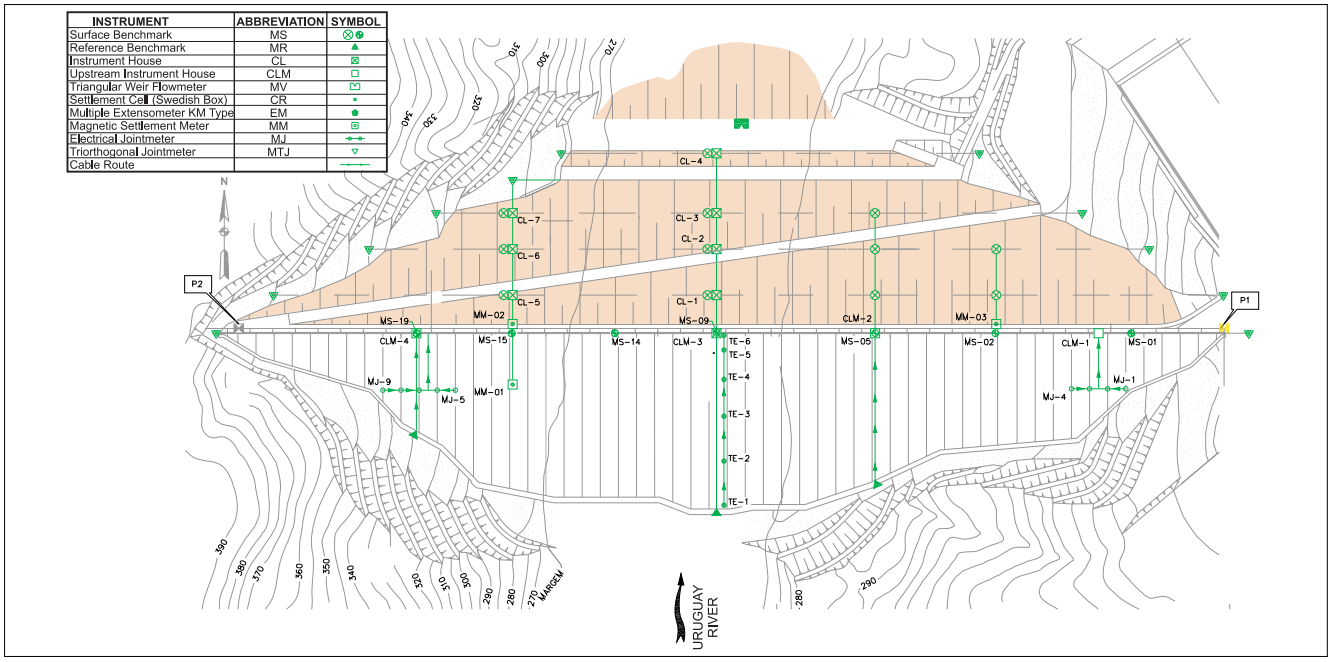

Figure 9 - Main Dam - Plan with Instrumentation

the upstream slope influenced the displacement of the dam, mainly on the upstream slope where the direction of displacement in the bottom part of this slope was modified. The largest displacement noted occurred in the electro-level EN-9, which is installed at El. 309.00.

At the end of the construction (December 1999) the displacement observed in EN-9 was 12 cm in the upstream direction, but by the end of the filling this displacement inverted direction and showed 45 cm displacement towards downstream. Figure 10 below shows the displacements, in centimetres, perpendicular to the face of the concrete slab upstream, during the filling of the dam.

Figure 11 illustrates the horizontal displacements and settlements of the parapet wall of the main dam measured by the surface benchmarks.

The seepage rates through the Itá dam were first recorded when the reservoir level reached around 90 m from its maximum height. When the seepage rates monitored by the flow meter exceeded the 200 l/s mark, investigations were started in order to diagnose the possible causes of this occurrence. The actions taken to find the most suitable methodologies to solve the problem began when the flows exceeded the level allowed as established by the designer in the Operating Handbook of Instrumentation of the Itá HPP Structures (Qallowed = 500 l/s).

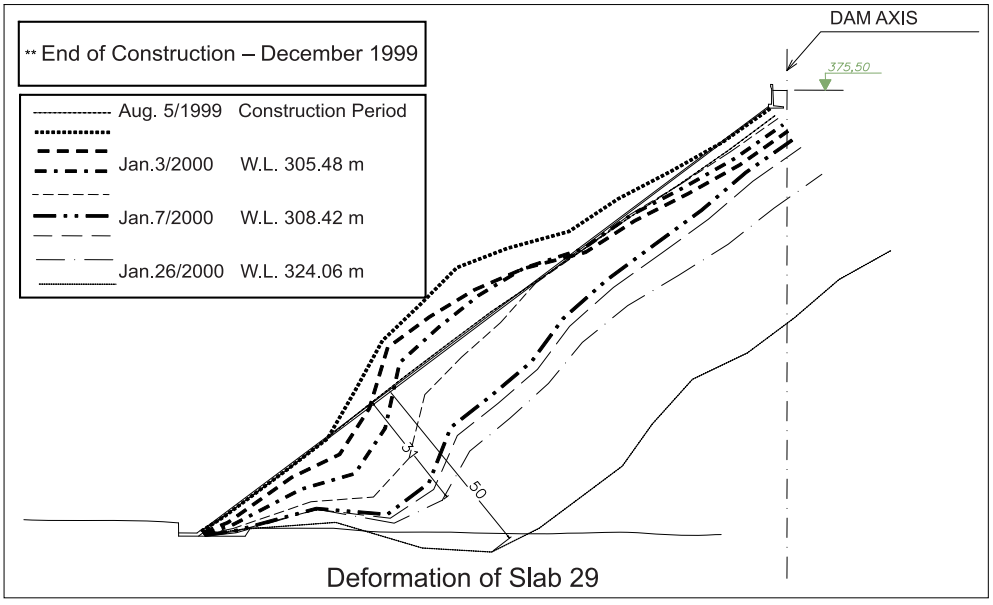

Figure 10 - Displacements Perpendicular to the Slab (cm)

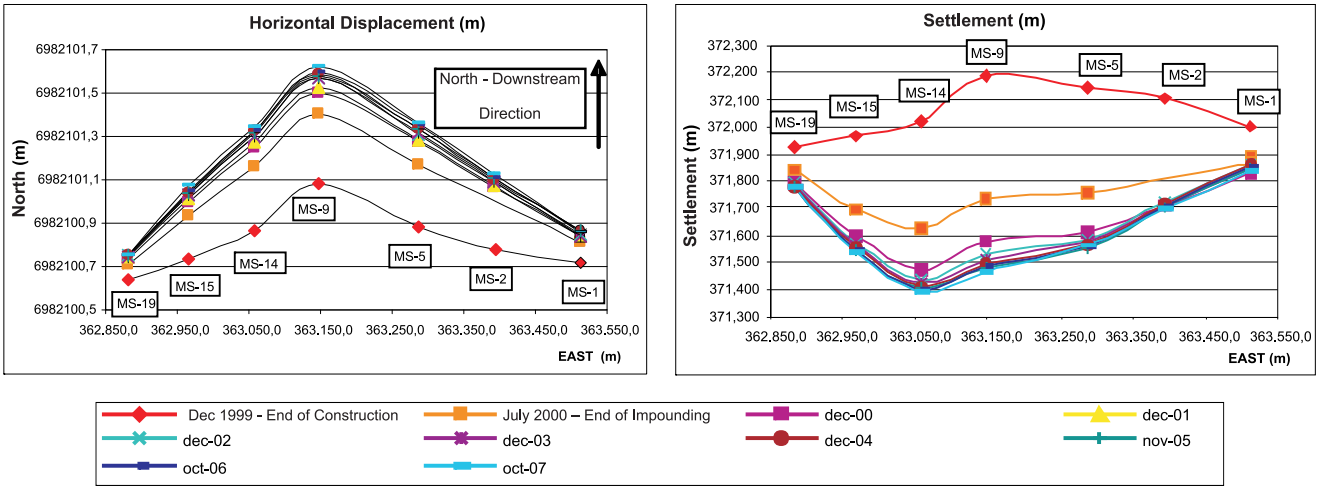

Figure 11 - Benchmarks of the Parapet Wall of the CFRD

The techniques used to detect the seepages through the face and dam foundation began with underwater inspections using divers. These inspections looked to investigate the accessible zones as far as 30 m in depth, and monitoring of the entire perimetral joint and of vertical joints between slabs was carried out using hydrophones. At this stage fluorescent tracers were also used to identify preferential paths of seepage through the dam foundation embankment, and piezometers were installed in the rock mass between the dam and the main spillway. In the sequence, underwater inspections were carried out using a controlled radio robot that identified, recorded and provided images for mapping the cracks occurring in the dam slab.

Figure 12 shows the face of the dam with the mapping of the cracked region, principally on the right bank, as the maps drawn in April and August 2000 show.

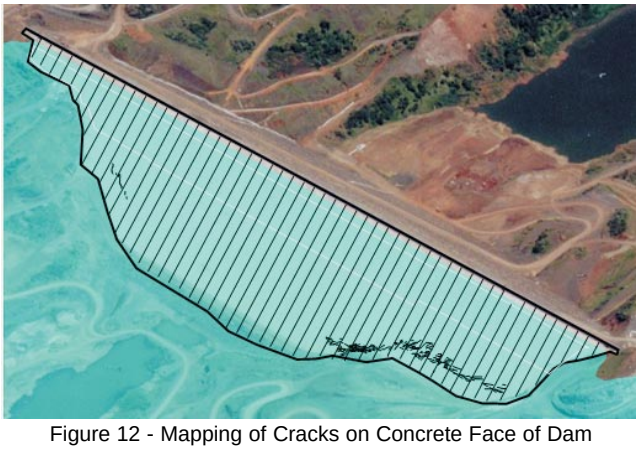

Figure 12 - Mapping of Cracks on Concrete Face of Dam

In April 2000 the seepage flows reached values of 1,700 l/s, when the effective dumping of sandy materials began from the dam top.

In the first stage of treatment between April and June 2000, around 1,800 m³ of artificial sand was laid and 4,400 m³ of clay-sandy silt dumped from the top of the dam over the slabs. All joints with seepage to the depth of 50 m were treated with the help of divers by pouring a mix of cement and sand with trace 1:16.5 in weight, having successfully staunched the seepage flows by dumping 0.01 to 4.0 m³ at each seepage point.

In depths of more than 50 m, pouring a sand-clay silt material in the slabs L8 and L9 reduced the flow to around 230 l/s, while in slab L20, by laying artificial sand, the drop in seepage rates was 900 l/s, showing the good performance of the two types of material used in the treatment.

During September 2000 silt was laid in the region of slabs L13, L14 and L19 to achieve a reduced flow of 200 l/s. From that date an ongoing reduction in the seepage rates through the CFRD was noted, practically without significant influence of another two launchings of granular materials at later periods.

Currently, the seepage value recorded by the system that collects and measures the seepage flows through the body of the Itá CFRD is 110 l/s, which are below that recorded at the end of the treatments. Figure 13 illustrates the course of seepage flows over the years.

9. TECHNICAL FEATURES

General

Location	Uruguay River, between towns Itá (SC) and Aratiba (RS)
Year of start	March 1996
Year of finish	June 2000
Proprietor	Itá Energética S.A. - ITASA & GERASUL - Centrais Generadoras do Sul do Brasil S.A.
Designer	Engevix Engenharia S.A.
Contractor	CBPO (civil works)
Erection	TENENGE
Manufacturers - electromechanical equipment	ABB/ ALSTOM/ VOITH/ COEMSA-ANSALDO/ BARDELLA

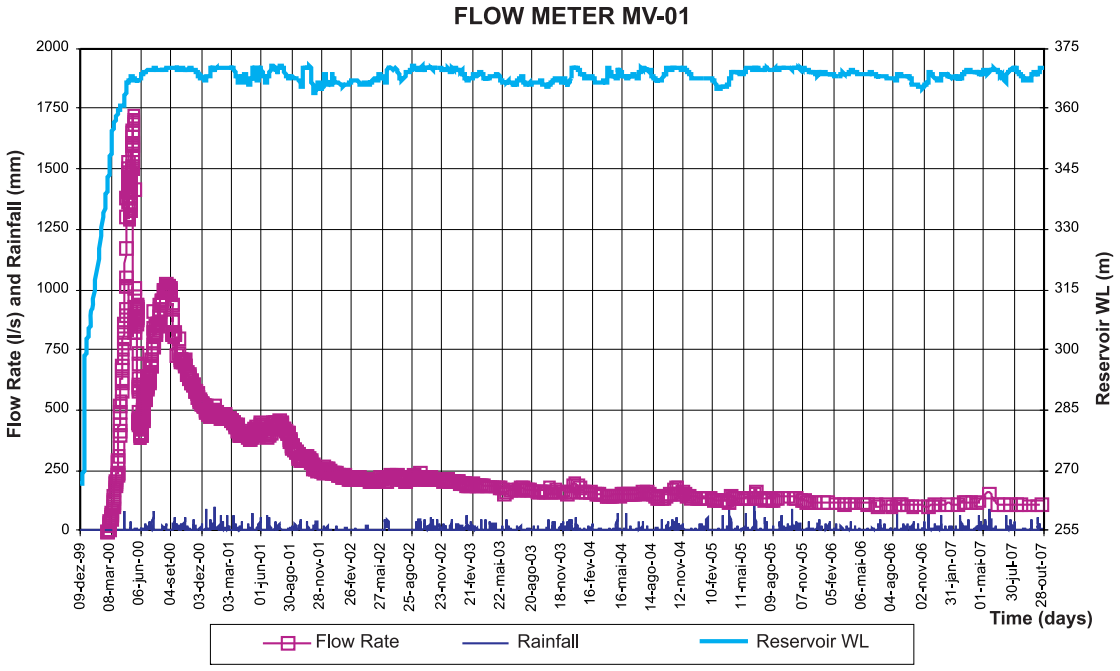

Figure 13 - Seepage Flows Measured by Flow Meter MV-01

Basic data

Drainage basin area	44,500 km²
Annual average rainfall	1,900 mm
Annual average temperature	18 ºC

Reservoir

Area at normal maximum level	141 km²
Total storage volume	5,100 x 10^6 m³
Active storage volume	750 x 10^6 m³
Normal maximum water level	370.00 m
Exceptional maximum water level	375.70 m
Minimum water level	364.00 m

Tailrace channel

Normal maximum water level	265.05 m
Maximum flood water level	291.40 m
Minimum water level	262.45 m

Flows

Average inflow	1,080 m³/s
Maximum flow recorded	29,620 m³/s
Minimum daily flow recorded	114 m³/s
Maximum diversion flow & recurrence interval	19,520 m³/s with Tr =10 years
10,000 year maximum flood inflow	52,800 m³/s

Dam

Type	concrete-face rockfill
Length	880 m
Height	125 m
Crest Elevation	375.50 m
Crest Width	7.0 m

Spillway

Type	surface
Length	39.00 m (main) & 54.00 m (auxiliary)
Width	130.50 m (main) & 85.50 m (auxiliary)
Capacity	49,940 m³/s

Spillway Gates

Type	radial
Quantity	10
Dimensions	18 m x 20.00 m
Width	20.00 m
Height	18.00 m
Manufacturer	COEMSA-ANSALDO

Intake

Type	Hollow gravity
Length	88.80 m
Maximum height	35.50 m

Intake gates

Type	fixed wheel
Quantity	5
Dimensions	7.20 m x 9.30 m
Width	7.20 m
Height	9.30
Manufacturer	COEMSA-ANSALDO

Diversion

Type	through 5 tunnels
Diversion structure	concrete
Height	32.00 m
Length	40.00 m

Diversion tunnel

Quantity	5
Type & dimensions of section (tunnels 1 & 2)	horseshoe (14 m x 14 m)
Length	566.00 m
Type & dimensions of section (tunnels 3, 4 & 5)	horseshoe (15 m x 17 m)
Length	530.00 m

Penstock

Type	tunnels
Quantity	5
Inner diameter	6.80 m
Length	18.35 (steel lined stretch)

Powerhouse

Type	indoor
Height.	56.00 m
Length	120.50 m
Installed capacity	290 MW

Turbine

Type	Francis
Quantity of units	5
Rated capacity	294.4 MW
Rated head	102.00 m
Maximum outflow of unit	313 m³/s
Rated velocity	128.57 rpm
Manufacturer	VOITH

Generator

Type	synchronous, three-phase, vertical axis
Rated capacity	305 MVA
Voltage	16 kV
Frequency	60 Hz
Rotation	128.57 rpm
Manufacturer	Ansaldo Italiana

Step-up transformers

Quantity	5 + 1 reserve
Type	three-phase, immersed in insulating oil
Rated capacity	305 MVA
Voltage	16 kV
Manufacturer	Ansaldo- ABB

Quantities

Common excavation	9,600,000 m³
Rock excavation	8,000,000 m³
Underground excavation	642,000 m³
Rockfill and Transitions	11,110,000 m³
Concrete (conventional)	504,000 m³
Steel	30,000 ton
Cement	115,000 ton

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